DAMAGED-BASED LIFE OF STRUCTURES IN SEISMIC ENVIRONMENT

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INDIAN INSTITUTE OF TECHNOLOGY KANPUR

JUNE, 1997

DAMAGE-BASED LIFE OF STRUCTURES IN SEISMIC ENVIRONMENT

A Thesis Submitted

in Partial Fulfilment of the Requirements

for the Degree of

MASTER OF TECHNOLOGY

by

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to the

DEPARTMENT OF CIVIL ENGINEERING

INDIAN INSTITUTE OF TECHNOLOGY KANPUR

June 1997

CERTIFICATE

It is certified that the work contained in the thesis entitled "**Damage-Based Life of Structures in Seismic Environment**" by "**R. Pradeep Kumar**" has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

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ACKNOWLEDGEMENTS

I thank my advisor, Dr V.K. Gupta for his inspirational guidance and support during the entire tenure of my thesis period. The memories of this period will remain with me for ever. Apart from the academic support throughout my stay at IIT, Kanpur, I would also like to place on record my gratitude to Dr Gupta for the development of my overall personality.

I am greatly thankful to Biswajit Basu for providing all the support in regard to the discussions of technical content of the thesis

I am thankful to all of my teachers at IIT, Kanpur, from whom I have learnt a lot for my future professional career.

I am grateful to all of my friends at IIT, Kanpur who made my stay comfortable and memorable

R. Pradeep Kumar

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ABSTRACT

The current methodology for the aseismic design of structures is largely based on the concept of ensuring structural safety during a single earthquake event This event is usually the most severe event, during which availability of certain minimum ductility is ensured in the structure This design methodology does not consider the occurrence of several not-so-severe earthquakes and damage accumulation during each of these earthquakes, as may be necessary in the areas of moderate to high seismicity, and thus the actual life of the structure may sometimes be significantly less than the design life. In this study, a new approach has been proposed for the estimation of the actual design life of a single-degree-of-freedom (SDOF) structure in a given seismic environment such that at the end of this life, the structure is expected to attain a specified level of damage This approach is based on the estimation of expected number of earthquakes at each source with the help of Gutenberg-Richter relation and time-dependent hazard model, and on the estimation of structural damage during each of these events. The estimation of structural damage is based on the use of scaling models for Fourier spectrum, peak ground acceleration, and strong motion duration, on finding the properties of the equivalent linear SDOF oscillator, and on using the damage model proposed by Park and Ang. The structure is assumed to be hysteretic, elasto-perfectly plastic oscillator, and the strength and stiffness of this oscillator are assumed to degrade during each damaging event. A case study has been carried out for illustrating the proposed model, and it has been shown how this may be used to determine the design force levels for maximum allowable damage at the end of the design life.

CHAPTER I

INTRODUCTION

1.1 General Introduction

The earthquake-resistant design of structures according to the existing design philosophy aims to ensure that during their lifetime, the structures resist the maximum possible earthquake without collapse. The magnitude of this earthquake is estimated based on the available statistical data on past earthquakes. If the past data is not available, fault rupture parameters are used to assess this magnitude, or in other cases, it is based on the expert judgement. The epicentral distance of the fault at which this earthquake occurs is normally taken as the closest distance of this fault from the site. By using the estimated values of magnitude and epicentral distance with an attenuation relationship, peak ground acceleration (PGA) is estimated and the structure is designed for the level of forces consistent with this PGA. This design approach based on the single earthquake of largest magnitude is termed as the 'scenario earthquake' approach. This approach may be unsafe in those areas where several earthquakes of smaller magnitudes are also likely to occur besides the event for which the structure has been designed. This may also be unacceptable when the structure should not suffer damage beyond a critical limit due to the heavy financial losses associated with interruptions in business activity. Due to the first possibility, the cumulative damage in the structure may become so large during the smaller magnitude earthquakes only that the structure may become unusable before its design life is completed.

For reliable evaluation of the design levels which will be consistent with the allowable damage at the end of the design life, it is necessary to account for the seismicity of the area in a comprehensive manner and to estimate the number of different magnitude events with proper spatial distribution around the site. It is also necessary to estimate damage due to each of these events and to get cumulative damage at the end of design life of the structure for all of these events

Seismicity represents the expected rate of occurrence of earthquakes with different magnitudes. Along a fault, this is generally obtained by studying the frequency-magnitude relationship using the available information on the past earthquakes (Gutenberg and Richter (1942)), or by fault rupture parameters.

Early analytical methods to determine the seismic hazard at a site were based on PGA and the return periods were calculated with respect to this parameter only. Housner (1969) studied the attenuation of PGA in several regions of the United States and presented his results graphically in terms of fault length. Esteva and Villaverde (1973) derived expressions for PGA and peak ground velocity (PGV) on the basis of accelerations reported by Hudson (1971, 1972a,b). Esteva (1974) gave a procedure to combine the geological, geophysical and all other non-statistical evidences for producing a mathematical model of seismicity of the given area Attempts have also been made by Crouse (1973), Hudson and Udwadia (1973) and Salt (1974) to predict analytically the characteristics of motions on different soils. Yegulalp and Kuo (1974) applied the theory of extreme value for estimating the probabilities that given magnitudes will be exceeded in given time intervals.

Classical methods of time-series analysis have been applied by many researchers (e g , see Knopoff (1964), Aki (1963), Vere-Jones (1970) and Shlien and Toksoz (1970)) attempting to devise analytical models for random earthquake sequences Most commonly applied stochastic models of seismicity assume that events of earthquake occurrence constitute Poisson process and that the magnitudes are independent and identically distributed. Some drawbacks of this model become evident in the light of statistical formulation and the analysis of the physical process involved. The Poisson assumption implies that the distribution of the waiting time to the next event is not modified by the knowledge of the time elapsed since the last event, while the physical models of gradually accumulated and suddenly released energy call for a more general renewal process such that the expected time to the next event decreases with time (Esteva (1974)). Statistical data shows that Poisson assumption may be acceptable when dealing with the large shocks through out the world (Ben-Maneham (1960))

Statistical analysis of waiting times between earthquake does not favour the adoption of the Poisson model or of other forms of renewal processes Some of these forms assume that the waiting times are mutually independent with lognormal or Gamma distributions (Shlien and Toksoz (1970)). Therefore, one-step memory models or other semi-Markov models given by Patwardhan et al. (1980) and Kiremidjian and Suzuki (1987) with a time-dependent hazard rate such as the Weibull or the lognormal, have been used to evaluate the probabilities of occurrence Other models which have been developed (e.g., see Vere-Jones (1970)) are of the trigger type, i.e., the overall process of earthquake generation is considered as the superposition of number of time series, each having a different origin, where the origin times are the events of Poisson process.

Quantification of damage in structures due to earthquake loading is facilitated by the evaluation of damage index. The damage indices are defined locally for each element and are then integrated globally for the entire structure. Generally, the damage indices are considered to be dimensionless parameters and are intended to range between zero for an undamaged structure and one for fully damaged or collapsed structure, with an intermediate value giving a measure of the partial damage.

The two earliest and simplest damage indicators were ductility and interstory drift Despite the limitations they have with respect to the effect of cyclic loading, they are most widely recognized as the damage parameters because they are easy to calculate and their physical meaning is simple. Zhu et al. (1988) studied the effect of peak ground acceleration to velocity ratio, a/v, on ductility of inelastic systems. They discussed the significance of a/v ratio as the parameter for the ground motion characterization from a seismological point of view based on their damaging capacity. Stephens and Yao (1987) studied the cumulative damage in the structure using displacement ductility In this case, the damage index was used to assess two test structures which showed a moderate correlation with observed damage

As an improvement over the ductility ratio, Banon et al (1981) proposed flexural damage ratio which accounted for the stiffness and strength degradations that occur under cyclic loading. Roufaiel and Meyer (1987) improved upon the model of Banon et al. (1981) as it was not consistent with the test data. They defined flexural damage ratio as the increase in flexibility at maximum deformation divided by the increase in flexibility at failure. This parameter showed a

good correlation with the residual strength and stiffness of the specimens tested in flexure.

Accumulation of damage under cyclic loading is usually considered either by a low-cycle fatigue formulation in which damage is taken as a function of the accumulated plastic deformation or a term related to the hysteretic energy absorbed during the loading Fajfar (1992) proposed equivalent ductility factors based on damage due to low cycle fatigue, and used those for the construction of inelastic spectra Chung et al (1987, 1989) used the fatigue formulation for finding strength loss due to each cycle of loading. They calculated the damage index by using Miner's rule slightly modified by some weights which were based on the average stiffnesses for specimens cycled to failure. Wang and Shah (1987) developed a model based on the concept of accumulated damage Their model is capable of predicting the hysteretic behaviour of reinforced concrete beam-column joints, especially the strength and stiffness deterioration and the damage state of the member

Energy absorption was used by Gosain et al. (1977) as a measure of damage. He calculated damage as a cumulative energy ratio based on the assumption that if peak force has dropped below 75% of the yield value, the remaining capacity becomes negligible. Park and Ang (1985) proposed a model based on the premise that the total damage due to earthquake motion is linear combination of the damage caused by maximum deformation and the absorbed hysteretic energy. This model has been the most widely used model primarily because it has been calibrated for several damaged structures during the past earthquakes (see Park et al. (1987)).

Anderson and Bertero (1991) showed that large amplitude acceleration may not always cause appreciable damage by driving the structure into the non-linear range. In fact, many repetitive non-linear excursions of relatively smaller amplitudes may be more damaging during an earthquake of larger duration. Jeong and Iwan (1988) have studied the effect of earthquake duration on the damage of the structures by using Miner's rule. They calibrated the model against observed failures of reinforced concrete columns. Basu and Gupta (1995) proposed a probabilistic model to estimate the damage to the structure with a given ductility by using the order statistics of the higher order peaks. This model relates the damage to the entire response process, and not just to the largest response. Basu and Gupta (1996) proposed a stochastic technique for developing damage-based inelastic spectra for aseismic design of the structures which can be idealized as SDOF oscillators. For this purpose, the oscillators have been approximated by equivalent linear oscillators.

Fajfar and Gašperšič (1996) proposed a non-linear method called N2 method for the aseismic design of reinforced concrete buildings. This method is mainly applicable to the structures predominantly vibrating in the first mode.

This study proposes a new approach for the estimation of the design life of a SDOF structure by assuming time-dependent hazard model with log-normal distribution for the return period, and by considering the model of Park and Ang (1985) for the estimation of damage during a single event. The ground motion for each earthquake event has been characterized by using the scaling models developed by Trifunac and co-workers (Trifunac and Brady (1975, 1976), Trifunac and Lee (1985)).

1.2 Organization

This work is organized in three chapters following this chapter.

In Chapter II, the proposed approach has been formulated for the evaluation of the total damage expected to take place in a SDOF structure over a given design life while the structure is situated in an area of known seismicity The information on seismicity has been used to find the number of earthquakes in each magnitude range which are likely to occur at a fault during the design life of the structure. For a particular earthquake event, the ground motion characteristics at the site of the structure have been estimated by using the available scaling relationships, and the corresponding structural damage has been estimated by considering oscillator to be hysteretic in nature. The cumulative damage during the design life has been estimated by considering the most critical sequence of earthquake events and by considering 'event to event' stiffness and strength degradation for the oscillator.

In Chapter III, the proposed approach has been illustrated by considering a hypothetical area having four faults with different activity rates. The growth of damage in the structure with its age has been studied, and a parametric study has been carried out to highlight the interdependence of various parameters which govern the design life of a structure for a given maximum damage, in case of oscillators of different initial periods.

A brief summary and conclusions of this study have been presented in Chapter IV.

CHAPTER II

FORMULATION FOR DAMAGE-BASED LIFE PREDICTION

2.1 Brief Overview

We are interested in obtaining the life of a structure at a site located in an area of known seismicity for a permissible cumulative damage in the structure. The structure is expected to be subjected to certain numbers of earthquakes of different magnitudes in this period which are consistent with the seismicity of the area. The number of earthquake events of different magnitudes which are expected to occur in the area over a specified period from various nearby faults may be estimated by using available Gutenberg-Richter relationship and time-dependent hazard rate. The ground motion at the site under consideration during a particular event may be characterized for a level of confidence in form of the PSDF Even though scaling equations are available for directly estimating this PSDF (De (1997)), it is also possible to use the available scaling equations on Fourier amplitude spectrum (Trifunac and Lee (1985)), strong motion duration (Trifunac and Brady (1975)), and PGA (Trifunac and Brady (1976)) for this purpose. Assuming the ground motion process to be a stationary process, its PSDF may be estimated from the knowledge of Fourier amplitude spectrum and strong motion duration (Bendat and Piersol (1986)) and then, to account for non-stationarity, this PSDF may be scaled up or down uniformly at all frequencies so as to correspond to the same expected PGA as the PGA estimated by the regression relationship (Shrikhande and Gupta (1997)). Since the structures are designed to undergo significant inelastic deformations during the most severe

and moderately severe earthquakes, it is necessary to account for the non-linear behaviour of structure in estimating the damage during each event. For this purpose, stochastic linearization technique may be conveniently used to find the equivalent linear system properties in case of SDOF systems. Damage caused by each event may be then estimated by using a suitable damage model and information on ordered peak amplitudes (see, e.g., Basu and Gupta (1995)). By assuming a sequence of various events (as expected during the design life of the structure) which causes maximum cumulative damage, the design life may be predicted for a specified critical damage after an iterative process.

2.2 Seismicity and Expected Earthquake Events

For the calculation of cumulative damage due to various seismic events, it is necessary to know the number of events of different magnitudes occuring in a given time. For this, we need to know the rates of occurrences of the earthquakes of different magnitudes for each contributing source.

Let the magnitude, M_k be representative of all the earthquakes with magnitudes lying in a small interval, $(M_k - \Delta M/2,\ M_k + \Delta M/2)$, centered at M_k. Let R_l denote the epicentral distance of the lth source from the site under consideration. The average rate of occurrence per year, n_{lk}, of magnitude, M_k, earthquakes at the lth source may be obtained by using the following relationship given by Gutenberg and Richter (1942),

$$\log n_{lk} = a_l - b_l M_k \ . \tag{2.1}$$

Here, a_l and b_l are the constants estimated from the data of past earthquake records or from the known slip rate along the lth source. If all the events cor-

responding to a magnitude class at a source are assumed to follow a Poissonian sequence of occurrence, $1/n_{lk}$ would represent the mean return period of this occurrence for M_k magnitude earthquakes at the lth source However, it may not be realistic to assume a uniform rate of occurrence for the earthquake events over a given period of time. The likelihood of occurrence increases with the increase in the quiscent period, i.e., the period of strain build-up following an earthquake event. Hence, the return period should be distributed based on how much time has elapsed since the last event occurred on the source under consideration. Such a distribution follows from the assumed distribution of the return period by conditioning on the fact that no event has occurred till the instant of time under consideration. Thus, if $f(t)$ represents the probability density function of the return period, and $F(t)$ represents the corresponding cumulative probability distribution, the hazard rate,

$$h(t) = \frac{f(t)}{1 - F(t)} \tag{2 2}$$

would represent the probability of the event occurring in the time interval, $(t,\ t+dt)$ on the condition that no event has occurred in the preceding t years. However, since the Poissonian distribution is a memoryless distribution, its use in Eq. (2.2) will not be able to model the quiscent period. We can alternatively consider the return period to be lognormally distributed such that for a given magnitude interval, the median of the (lognormally distributed) return period is equal to the expected value of the exponentially distributed return period as in the Poissonian model (see Todorovska (1994)). Thus, the hazard rate in case of kth magnitude class and lth source may be written as

$$h_{lk}(t) = \frac{\phi\left[\frac{\ln t - \lambda_{lk}}{\zeta_{lk}}\right]}{\zeta_{lk} t \left(1 - \Phi\left[\frac{\ln t - \lambda_{lk}}{\zeta_{lk}}\right]\right)} \tag{2.3}$$

where, λ_{lk} and ζ_{lk} respectively are the mean and standard deviation of the random variable, $\ln T_{lk}$, where, T_{lk} denotes the return period for the kth magnitude event on the lth source. Further, $\phi(.)$ and $\Phi(\,)$ are the standard normal density and cumulative distribution functions. Due to the constraint as assumed by Todorovska (1994) and since the median of T_{lk} is equal to $e^{\lambda_{lk}}$, λ_{lk} in Eq. (2.3) becomes equal to $-\ln(n_{lk})$. Further, as suggested by Todorovska (1994), $\zeta_{lk} =$ 0 2 may be considered to be a reasonable estimate. It may be noted that the distribution of the return period, T_{lk}, is based on the quiscent period with respect to the event of magnitude, M_k, only It does not preclude the possibility of the occurrence of the earthquakes belonging to the other magnitude classes before the occurrence of the M_k magnitude earthquake.

If there are no events during the time of T_0 years since the last event, the number of occurrences of the M_k magnitude events on the lth source in Y years is given by

$$n_{lk}(T_0 + Y|T_0) = \int_{T_0}^{T_0+Y} h_{lk}(\tau)\mathrm{d}\tau \,. \tag{2.4}$$

2.3 Estimation of PSDF

For estimating the damage due to all the events as predicted by Eq (2 4), it is necessary to characterize the ground motion in terms of PSDF of the ground acceleration process for each event. This study considers the estimation of PSDF by using the known scaling relationships for Fourier spectrum, strong motion duration, and PGA in terms of the parameters like earthquake magnitude, epicen-

tral distance and geologic site conditions For the scaling of Fourier spectrum, the following scaling relationship as proposed by Trifunac and Lee (1985) has been considered,

$$\log_{10} FS(T) = M + \mathcal{A}tt(\Delta, M, T) + b_1(T)M + b_2(T)s + \frac{b_3(T)\Delta}{100} + b_4(T) + b_5(T)M^2 \quad (2.5)$$

Here, M is the earthquake magnitude, Δ is the representative distance from source to station, and s (= 0 for alluvium, 1 for intermediate, and 2 for rock) represents the site condition, for the desired combination of event and site The representative distance, Δ, is expressed in terms of the epicentral distance, R, focal depth, H, fault size, S, and coherence radius, S_0, as

$$\Delta = S\left(\ln\frac{R^2 + H^2 + S^2}{R^2 + H^2 + S_0^2}\right)^{-\frac{1}{2}} \quad (2\ 6)$$

Further, in Eq. (2 5), b's represent the coefficients determined from a regression analysis at each period, T, and $\mathcal{A}tt(\Delta, M, T)$ represents the attenuation function given by

$$\begin{aligned} \mathcal{A}tt(\Delta, M, T) &= \mathcal{A}_0(T)\log_{10}\Delta & R \leq R_0 \\ &= \mathcal{A}_0(T)\log_{10}\Delta_0 - (R - R_0)/200 & R > R_0 \quad (2.7) \end{aligned}$$

Here, R_0 is the epicentral distance from the earthquake source at which the surface waves start dominating the ground motion, Δ_0 is the representative distance corresponding to R_0, and $\mathcal{A}_0(T)$ is the empirically determined attenuation function

For the scaling of strong motion duration, T_s, following relationship given by Trifunac and Brady (1975) is proposed to be used,

$$T_s = -4.88s + 2.33M + 0\ 149R . \quad (2.8)$$

It may be mentioned that besides relating the Fourier spectrum amplitudes with the PSDF amplitudes, the strong motion duration plays a key role in determining the total number of cycles and thus the structural damage during the earthquake excitation (Basu and Gupta (1995)).

Assuming the earthquake ground motion to be a stationary process, the PSDF corresponding to the M_k magnitude event occurring at the lth source is calculated at frequency, ω, as

$$G_{lk}(\omega) = \frac{Z_{lk}^2(\omega)}{\pi T_{lk}} \tag{2 9}$$

where, $Z_{lk}(\omega)$ and T_{lk} respectively are the expected Fourier spectrum and strong motion duration for magnitude, M_k, and epicentral distance, R_l, as estimated from Eqs. (2 5) and (2.8).

To include the effect of non-stationarity of ground motion, $G_{lk}(\omega)$ is proposed to be scaled up or down so as to correspond to the same expected PGA as that estimated by a suitable scaling relationship in case of magnitude, M_k, and epicentral distance, R_l. For this purpose, it is proposed to use the scaling relationship given by Trifunac and Brady (1976) as follows

$$\log PGA = M + \log A_0(R) - \log a(M) \ . \tag{2.10}$$

Here, $A_0(R)$ represents the attenuation of acceleration with epicentral distance and $a(M)$ represents the magnitude-dependent scaling constant as determined by a regression analysis.

2.4 Equivalent Linear Oscillator and Response Statistics

Once the PSDF is determined for an event of magnitude, M_k, occur-

ring at the lth source, it is convenient to estimate the structural response by replacing the given non-linear oscillator by an equivalent linear oscillator. Once the properties of the equivalent oscillator are determined by applying a suitable linearization technique, the PSDF of its response may be obtained simply by multiplying the ground PSDF with the squared modulus of the response transfer function. The response PSDF may now be used to estimate the relevant response parameters for calculating damage.

Let us consider a hysteretic, SDOF structural system with initial frequency, ω_n, post-yield frequency of vibration, $\alpha\omega_n$, and viscous damping ratio, ζ. The equation of motion of this oscillator, when subjected to the base excitation, $\ddot{u}_g(t)$, may be written as

$$\ddot{x} + 2\zeta\omega_n\dot{x} + f(x,z) = -\ddot{u}_g(t) \tag{2.11}$$

where, $\ddot{x}$ and $\dot{x}$ denote the mass acceleration and velocity respectively. Further, $f(x,z)$ denotes the acceleration term corresponding to non-linear hysteresis modelled as the combination of linear and elasto-plastic system. This is expressed as (Suzuki and Minai(1987))

$$f(x,z) = \alpha\omega_n^2 + (1-\alpha)\omega_n^2 z \tag{2.12}$$

where, z is obtained from the following differential equation,

$$\dot{z} = \dot{x}\left[1 - U(\dot{x})U(z-x_y) - U(-\dot{x})U(-z-x_y)\right] \tag{2.13}$$

In Eq. (2.13), $U(\,)$ denotes the Heaviside step function and x_y represents the yield displacement for the oscillator. By statistical linearization (see, e.g., Roberts and Spanos (1989)), the above non-linear oscillator can be replaced by an equivalent

linear oscillator with damping ratio, ζ_e, and natural frequency, ω_e. These parameters can be evaluated by averaging the response over one cycle as proposed by Caughey (1960), and thus, we have

$$\omega_e^2 = \omega_n^2 - \omega_n^2 g(\sigma_y)(1-\alpha) \tag{2.14}$$

$$\zeta_e = \frac{\zeta\omega_n}{\omega_e} + \sqrt{2\pi}\frac{(1-\alpha)\omega_n}{\omega_e\sigma_y}\left[1 - \mathrm{erf}\left(\frac{1}{\sqrt{2}\sigma_y}\right)\right] \tag{2.15}$$

with

$$\sigma_y = \frac{x_{\mathrm{rms}}}{x_y}\ , \tag{2.16}$$

$$g(\sigma_y) = \frac{1}{2\pi\sigma_y^4}\int_1^\infty A^3\left(\pi - \Lambda - \frac{1}{2}\sin 2\Lambda\right)\exp\left(-\frac{A}{2\sigma_y^2}\right)\mathrm{d}A\ , \tag{2.17}$$

and erf(.) representing the error function. In Eqs. (2.16) and (2.17), we have

$$\Lambda = \cos^{-1}\left(1 - \frac{2}{A}\right) \tag{2.18}$$

and

$$x_{\mathrm{rms}} = \left[\int_0^\infty \frac{G_{lk}(\omega)}{[(\omega_e^2-\omega^2)^2 + (\zeta_e\omega_e)^2]}\mathrm{d}\omega\right]^{\frac{1}{2}}\ . \tag{2.19}$$

It may be noted that an iterative procedure is usually necessary for obtaining the values of ω_e and ζ_e. Thus, with an initial guess of σ_y, the values of ω_e and ζ_e are obtained by using Eqs. (2.14) and (2.15), and then σ_y is updated by using Eq. (2.16). After obtaining the linearized properties, ω_e and ζ_e, the response PSDF is calculated as

$$E_{lk}(\omega) = \frac{G_{lk}(\omega)}{(\omega_e^2-\omega^2)^2 + (\zeta_e\omega_e)^2}\ . \tag{2.20}$$

From the response PSDF, $E_{lk}(\omega)$, now, the expected amplitude of the ith order response peak, i.e., $E[x_{(i)}]$, may be estimated by using the order statistics approach as proposed by Gupta and Trifunac (1988). Thus, we have

$$E\left[x_{(i)}\right] = x_{\mathrm{rms}}\int_{-\infty}^{\infty}\eta p_{(i)}(\eta)\mathrm{d}\eta\ , \tag{2.21}$$

where

$$p_{(i)}(\eta) = \frac{N!}{(N-i)!(i-1)!}\left[P(\eta)\right]^{i-1}\left[1-P(\eta)\right]^{N-i} p(\eta) \qquad (2.22)$$

is the probability density function of the ith order peak. In Eq (2.22),

$$p(\eta) = \frac{1}{\sqrt{2\pi}}\left[\varepsilon e^{-\eta^2/2\varepsilon^2} + (1-\varepsilon^2)^{1/2}\eta e^{-\eta^2/2}\int_{-\infty}^{\eta(1-\varepsilon^2)^{1/2}/\varepsilon} e^{-x^2/2}\mathrm{d}x\right] \qquad (2.23)$$

and

$$P(\eta) = \int_{\eta}^{\infty} p(u)\mathrm{d}u \qquad (2.24)$$

respectively are the probability density and distribution functions of the peaks in the displacement response process, and

$$N = \frac{T_{lk}}{2\pi}\left[\frac{\lambda_4}{\lambda_2}\right]^{1/2} \qquad (2.25)$$

is the total expected number of peaks in this process. In Eq. (2 23), ε is the bandwidth parameter defined as

$$\varepsilon = \left[\frac{\lambda_0\lambda_4 - \lambda_2^2}{\lambda_0\lambda_4}\right]^{\frac{1}{2}} \qquad (2.26)$$

where λ_n is, in general, the nth moment of the PSDF, $E_{lk}(\omega)$, and is defined by

$$\lambda_n = \int_0^{\infty} \omega^n E_{lk}(\omega)\mathrm{d}\omega, \quad n = 0, 1, 2, \ldots \qquad (2.27)$$

2.5 Damage Model

For the N ordered peaks with amplitudes, $E[x_{(1)}]$, $E[x_{(2)}]$, , $E[x_{(N)}]$ as estimated in the previous section, the damage is proposed to be calculated as

per the model of Park and Ang (1985) This model consists of simple linear combination of normalized deformation and hysteretic dissipated energy as

$$D_{lk} = \frac{x_m}{x_u} + \beta \frac{EH}{Q_y x_u} \tag{2.28}$$

where, x_m is the maximum displacement that the equivalent linear SDOF system would be subjected to during the base excitation, x_u ($= \mu x_y$ where μ is the available ductility) is the ultimate displacement of the system under monotonic loading, β represents the effect of cyclic loading on structural damage, EH represents the total energy dissipation in the structure during the excitation, and Q_y is the yield strength of the structure. Following the modifications introduced in this model by Kunnath et al. (1992), following modified form of Eq (2.28) has been considered in this study

$$D_{lk} = \frac{x_m - x_y}{x_u - x_y} + \beta \frac{EH}{Q_y x_u} \tag{2.29}$$

In Eq (2.29), the first term is a simple, pseudo-static displacement measure. It takes no account of cumulative damage which is accounted for solely by the second term. The advantage of this model is its simplicity and the fact that it has been calibrated against a significant amount of observed seismic damage.

The parameter, x_m, in Eq. (2.29) refers to the expected largest peak amplitude, $E[|x|_{(1)}]$, in the absolute response process, $|x(t)|$. It may be easily estimated by following the same procedure as explained for $E[x_{(1)}]$ in the previous section. However, in this case, the probability density function, $p(\eta)$ and N are to be respectively replaced by $\tilde{p}(\eta)$ ($\eta \geq 0$) and $\tilde{N}$ $\tilde{p}(\eta)$ and $\tilde{N}$ may be expressed as (Gupta (1994))

$$\tilde{p}(\eta) = \frac{1}{\sqrt{2\pi}} \left[2\varepsilon e^{-\eta^2/2\varepsilon^2} + (1-\varepsilon^2)^{1/2} \eta e^{-\eta^2/2} \int_{-\eta(1-\varepsilon^2)^{1/2}/\varepsilon}^{\eta(1-\varepsilon^2)^{1/2}/\varepsilon} e^{-x^2/2} \mathrm{dx} \right] \tag{2.30}$$

and

$$\tilde{N} = \frac{T_{lk}}{2\pi}\left(1+\sqrt{1-\varepsilon^2}\right)\left[\frac{\lambda_4}{\lambda_2}\right]^{1/2} . \quad (2.31)$$

For reinforced concrete elements, β can be defined as a function of the value of shear and axial forces on the section and the total amount of longitudinal and confinement reinforcement. By means of regression analyses using large sets of cyclic test data, Park and Ang (1985) have suggested relationships for β. Many researchers (e.g., see Kunnath et al (1990), Stone and Taylor (1993)) have also proposed regression relations for finding β. This study considers $\beta = 0.1$ which has been considered as a default value in the software, IDARC (Kunnath et al. (1990))

Assuming the response to be a narrow-banded process, the total number of cycles in the response may be considered to be equal to the total number of positive zero crossings given by

$$N_0 = \frac{T_{lk}}{2\pi}\left[\frac{\lambda_2}{\lambda_0}\right]^{1/2} . \quad (2.32)$$

However, since the actual response process is not exactly a narrow-banded process, the total number of estimated peak amplitudes will be more than N_0 Hence, it is assumed that the largest N_0 only of those will contribute to the energy dissipation, EH. Thus, for $\alpha = 0$, the total energy dissipation is calculated by the following equation,

$$EH = \sum_{i=1}^{N_0} 4Q_y(E[x_{(i)}] - x_y) . \quad (2.33)$$

The structural damage during the event of magnitude, M_k, occurring at the lth source now follows from Eq. (2.29) as

$$D_{lk} = \frac{E[|x|_{(1)}] - x_y}{x_y(\mu - 1)} + \beta\frac{\sum_{i=1}^{N_0} 4(E(x_{(i)}) - x_y)}{\mu x_y} \quad (2.34)$$

It may be noted that the parameter, x_y or Q_y, is to be assigned suitable values depending upon the available ductility in the system. It is the common practice in the aseismic design to provide lower yield strength, Q_y, than what would be required for the elastic response during the most critical earthquake in the lifetime of the structure. Q_y is obtained by an iterative procedure in such a way that the maximum displacement of the (non-linear) structural system under the critical excitation is yield displacement, x_y, times the available ductility It may be noted that due to greater damage associated with greater reduction in Q_y (compared to the elastic strength), higher value of Q_y than that calculated for a given ductility may sometimes have to be chosen to limit damage to a specified level during the most critical event.

2.6 Cumulative Damage and Life Prediction

Once the expected damage for each event is estimated as explained in the previous section, the expected cumulative damage in the design life of the structure may be estimated by considering the occurrence of events of various magnitudes at each source as discussed in Section 2.2, and by adding the individual damage due to each of these events. It may noted that there will be no damage due to those events for which the expected maximum displacements do not exceed the yield displacement, x_y. Since the proposed seismic hazard model does not give any information about the sequence of the expected events during the design life of the structure and sequence is important for a stiffness and strength degrading structure, following approximate procedure is suggested to calculate the progressive damage during the design life

The expected number of events for magnitude, M_k, at lth source are estimated for first 5, 10, 15, .. years From this, the expected number of events in a particular block of 5 years is estimated by subtracting the number till the previous block from the number estimated till this block. For example, for the block starting at the 15 years of age of the structure, the expected number of events estimated for first 15 years is subtracted from that estimated for the first 20 years. Within each block, all the anticipated events are arranged in the decreasing order of the damage they would cause to the just-built, i.e., undamaged structure. This is done to maximize the estimated damage during the block under consideration. The cumulative damage at the end of each block is calculated by adding the damage due to all the events supposed to have taken place till that much age of the structure.

The present study considers the fact that after the occurrence of each damaging event, there will be degradation in the stiffness and strength of the structure. Following the logic that such degradation should be more in case of more damage and since no model is available yet to predict this 'event to event' degradation, following degradation model is proposed for this study.

It is assumed that the yield displacement, x_y, increases by Δx_y after each damaging event such that

$$\Delta x_y = \left(\frac{k_1 - k_{i+1}}{k_1 + k_{i+1}} \right) x_y \tag{2.35}$$

where, k_1 denotes the initial stiffness of the system and k_{i+1} denotes the stiffness of the system after the occurrence of the ith event. It may be noted that Eq (2.35) is based on the assumption that the maximum value of Δx_y is x_y in case of zero stiffness. Now, assuming that the stiffness degradation is an increas-

ing function of the maximum displacement beyond the yield displacement during an event, the stiffness, k_{i+1}, may be expressed as

$$k_{i+1} = k_i \left[1 - \frac{E[|x|_{(1)}] - x_y}{x_u - x_y}\right]^{\gamma} \tag{2.36}$$

where, the parameter, γ, controls the rate of 'event to event' degradation. This has been chosen arbitrarily as 0.1, considering the degradation in the moment capacity of the reinforced concrete members as modelled by Reinhorn et al. (1992) for a particular event. It may be noted that the strength degradation has been assumed to be governed by the stiffness degradation as in Eq (2 36) and by the increase, Δx_y, in the yield displacement as in Eq. (2.35).

The above procedure gives the estimate of expected cumulative damage at the end of the design life of the structure. By an iterative procedure, the design life of the structure can be easily predicted for a specified cumulative damage.

CHAPTER III

ILLUSTRATION OF THE PROPOSED APPROACH

3.1 Brief Overview

The approach proposed in Chapter II for the estimation of progressive increase in the structural damage with time and prediction of the design life of the structure for a specified damage level has been illustrated in this chapter. One hypothetical seismic environment with complete knowledge of the seismic activity along each fault has been considered for this purpose.

3.2 Site Data

The structure is assumed to be located in a hypothetical area consisting of four faults. Two faults are located at a distance of 30 km each from the site, while the other two faults are located at 40 and 50 km each. The values of a_l for these faults are taken to be 3.28, 4.09, 3.77 and 3.09 respectively while b_l has been assumed to be uniformly equal to 0.86 for all the four faults (Todorovska (1994)) The focal depths of the sources at all faults are assumed to be uniformly equal to 5 km The entire area is assumed to have alluvium geologic site condition.

3.3 Results and Discussion

Figs. 3.1 to 3.3 respectively show the progressive increase in the expected damage, as indicated by the index, D, with the age of the structure for the structure design life, $Y = 25$, 50 and 75 years. It is assumed in obtaining these

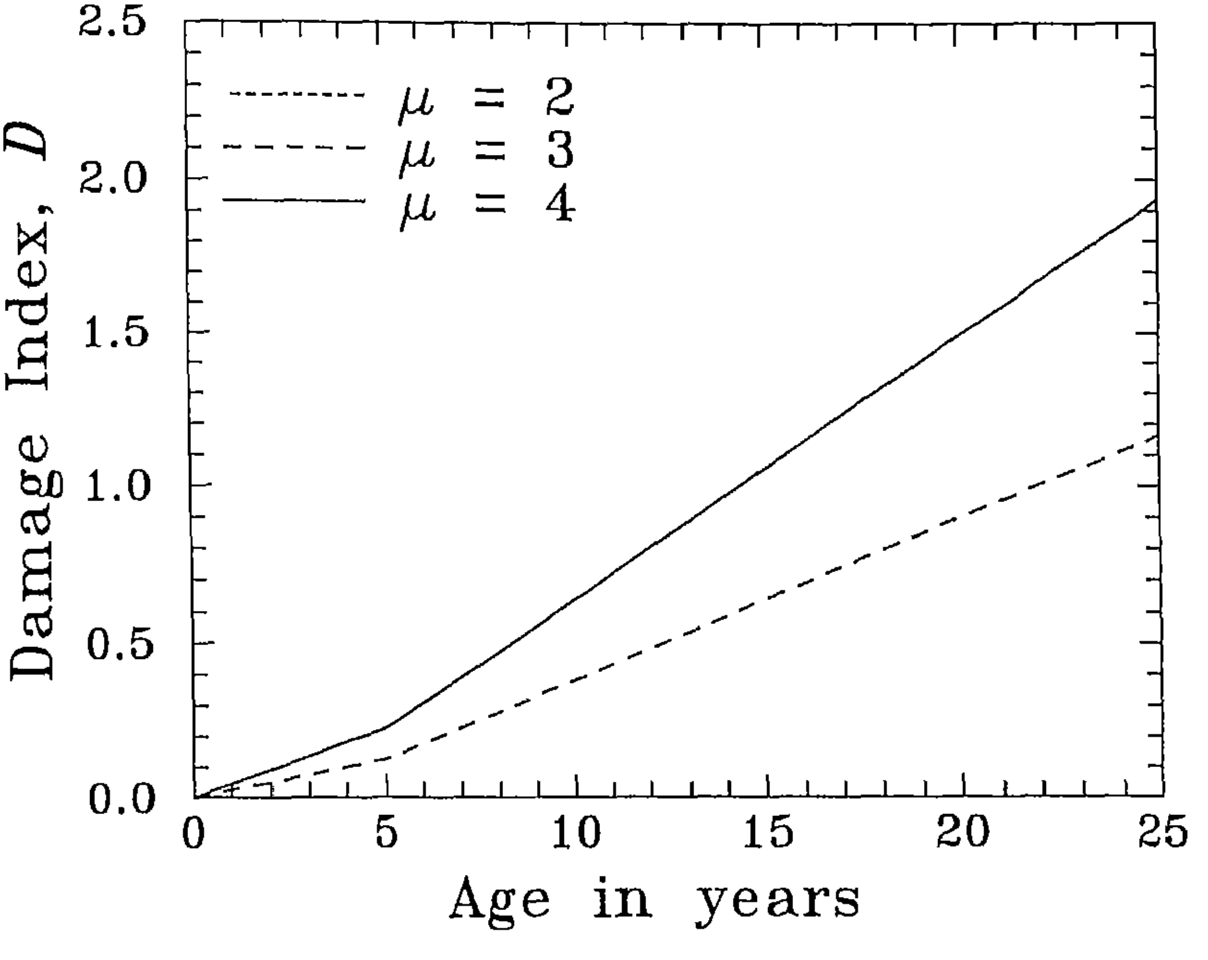

Figure 3.1 Progressive Damage for a SDOF Oscillator with 1 sec Period and $Y = 25$ years for $\alpha_R = 2.0$ and $\mu = 2$, 3 and 4.

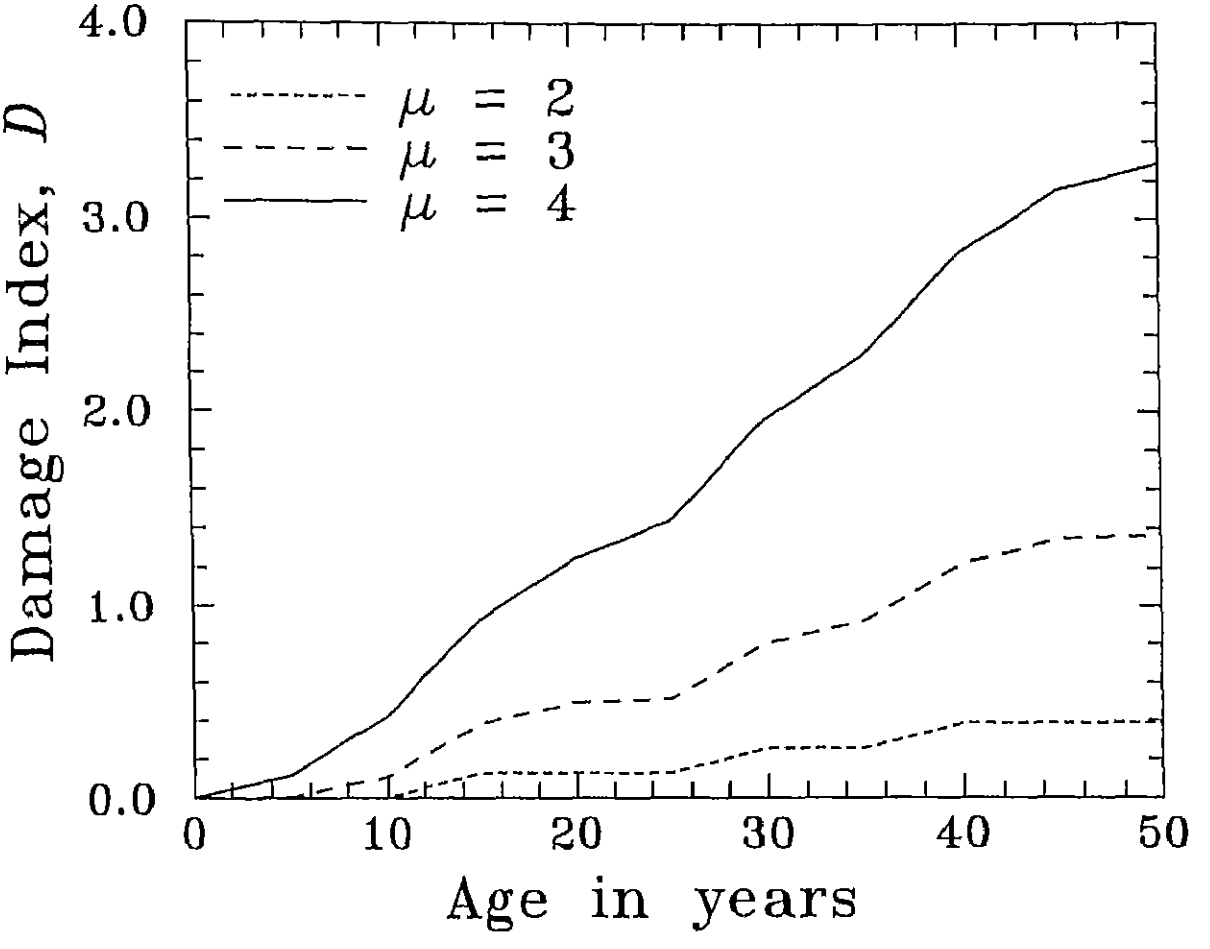

Figure 3.2 Progressive Damage for a SDOF Oscillator with 1 sec Period and Y = 50 years for α_R = 2.0 and μ = 2, 3 and 4.

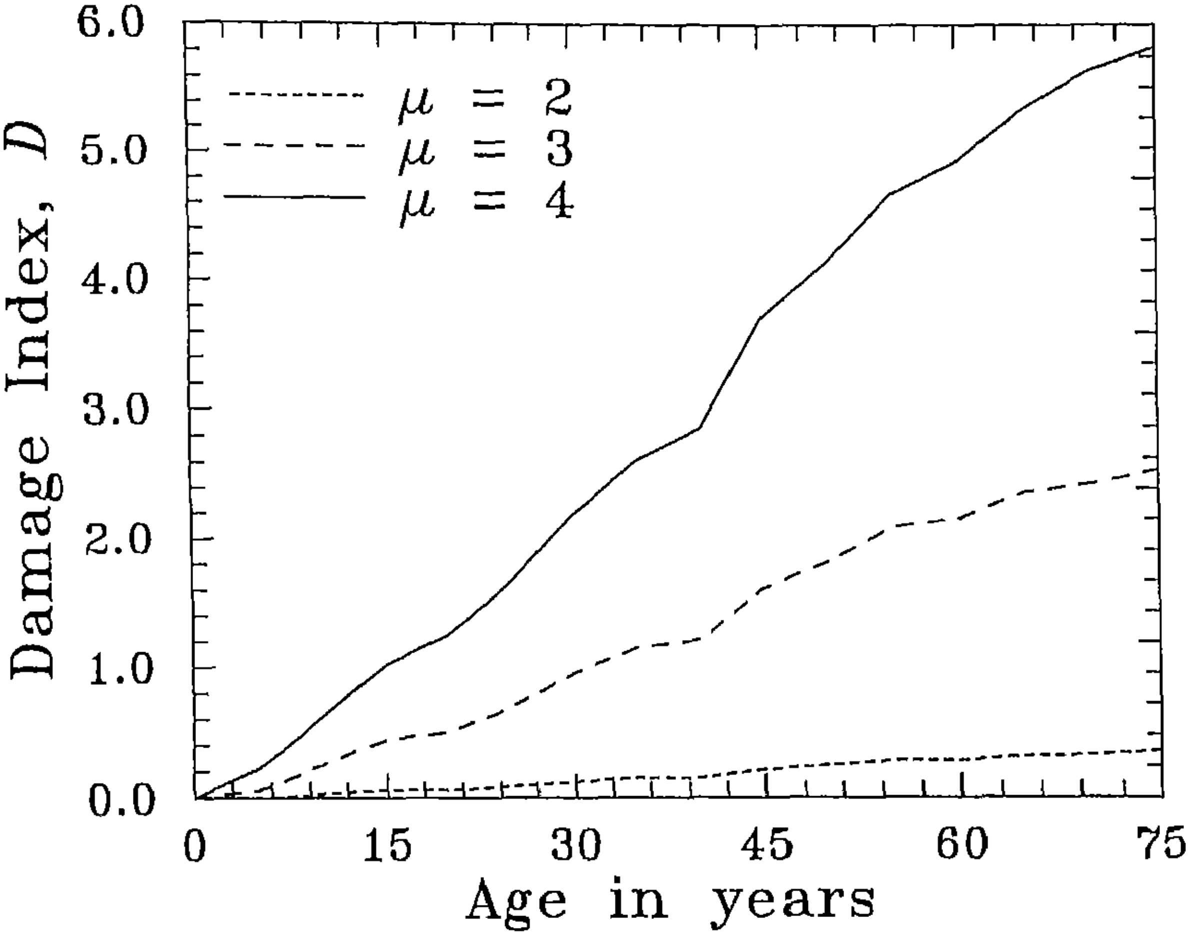

Figure 3.3 Progressive Damage for a SDOF Oscillator with 1 sec Period and $Y = 75$ years for $\alpha_R = 2.0$ and $\mu = 2$, 3 and 4.

figures that zero time has elapsed since the occurrence of last event on each fault, and that the total number of events expected during the design life are uniformly distributed over the various blocks of 5 years. Further, the structure has been assumed to have 1.0 sec period, 5% damping, $\alpha = 0$, and has been designed based on the largest magnitude earthquake that is expected to occur during its design life. The yield strength level has been fixed by taking $\alpha_R = 2$, where α_R denotes the ratio of the chosen yield strength level to the calculated yield strength level for which the maximum non-linear displacement during the most critical event is design ductility, μ, times the yield displacement In each figure, three curves corresponding to $\mu = 2$, 3 and 4 have been shown. For a given structure, the damage increases almost linearly with age which is consistent with the uniform temporal distribution assumed for various events It may be observed further that with the increase in μ, the expected damage as indicated by the damage index, D, increases in all the three cases Actually, with the increase in design ductility, the response reduction factors tend to become larger leading to greater reductions in obtaining the yield level of forces. In fact, in case of $Y = 25$ years, no damage is expected for $\mu = 2$ at any age of the structure because here, the response reduction factor is approximately equal to α_R, and thus, the structure behaves elastically during all events due to high yield strength level Similarly, in Fig. 3.2, the structures with $\mu = 2$ and 3 are not damaged at all for the first 5 and 10 years of age. Coming back to Fig. 3.1, it may observed that for $\mu = 3$ and 4, the structure reaches the critical damage level of 0.8 for collapse (see Ang et al. (1993)) well before the design life of 25 years. It reaches this level at the age of 18 and 12 years respectively. Thus, the structures with higher design ductility are likely to be critically damaged much before the end of design life. With the

increase in the design life also, the possibility of premature collapse appears to increase as indicated by Figs 3.2 and 3 3 Even though the design levels are higher in case of the structures with greater design life, the total damage is much higher for those due to the exposure to much greater number of damaging events. These trends will now be shown more clearly through a parametric study.

For the parametric study, 28 SDOF oscillators with the periods of 0.06, 0 08, 0.10, 0.12, 0.14, 0.16, 0 18, 0.20, 0 25, 0 30, 0.35, 0.40, 0 45, 0 50, 0.60, 0.70, 0.80, 0.90, 1.00, 1.20, 1 40, 1.60, 1.80, 2.00, 2 50, 3.00, 3 50, 4.00 have been considered, and the variations of cumulative damage, design life, design ductility, and the factor, α_R have been studied with the variations in the oscillator time period. In each case, two of the other parameters have been kept at their default values while the third parameter has been given different values. The default values of have been taken as: $\mu = 3$, $\alpha_R = 2$, $Y = 50$ years, and damage index $= 0.8$, while the different values given for obtaining different curves are: $\mu = 2$, 3 and 4; $\alpha_R = 1.5$, 2.0 and 2.5; $Y = 25$, 50 and 75 years; and damage index $=$ 0.4, 0.6 and 0.8.

Figs. 3.4 to 3.6 show the variations in the design life respectively for varying damage, design ductility, and α_R. Fig. 3.4 shows the possible design life values for different values of critical damages. It is seen that the structure should be designed for a lower design life if the damage is not allowed to cross a lower level It is also seen that for the allowable damage level of 0 8, the stiff oscillators can be designed for much greater life than the flexible oscillators while for the allowable levels of 0 4 and 0.6, both flexible and stiff oscillators have comparable design life values. It is so because the response reduction factors are lower for

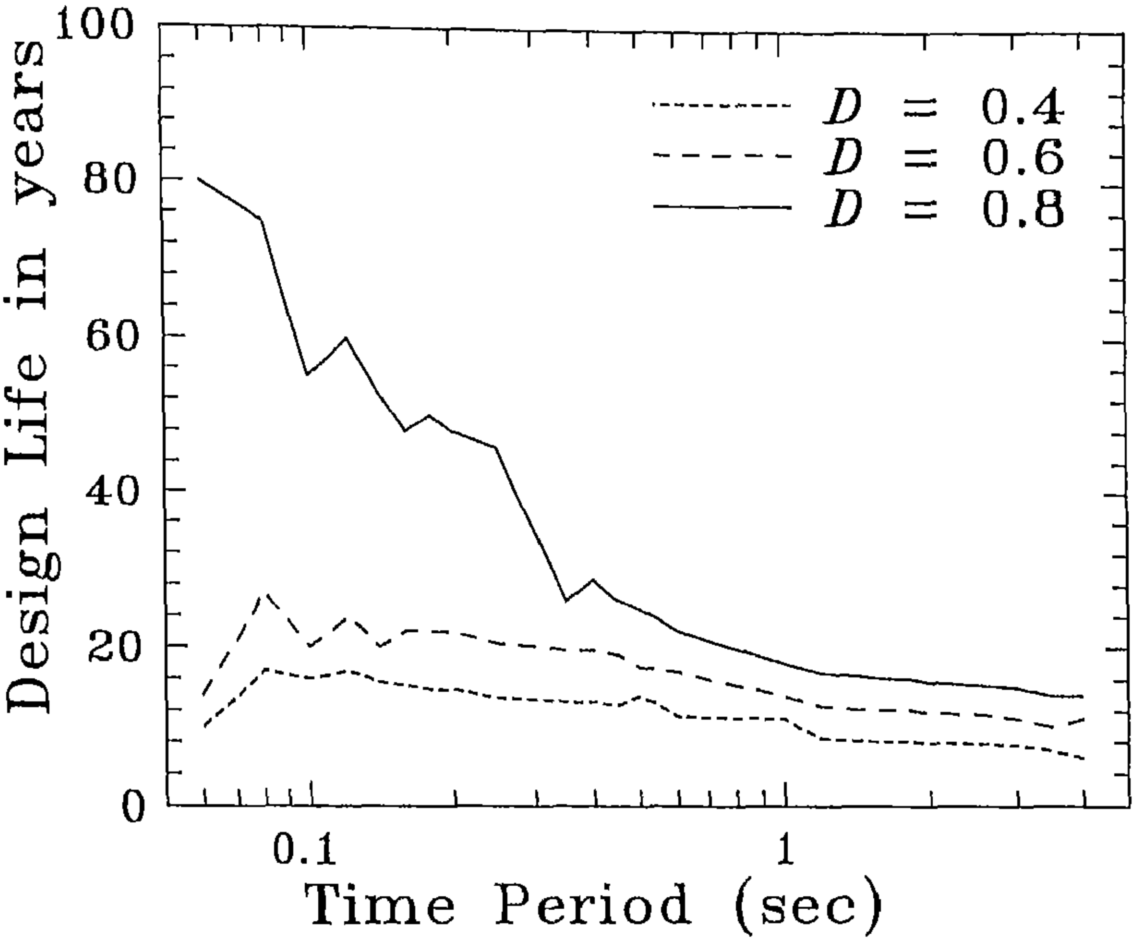

Figure 3.4 Design Life of a Set of Oscillators for an Allowable Damage, $D = 0.4$, 0.6 and 0.8, with $\alpha_R = 2.0$ and $\mu = 3$.

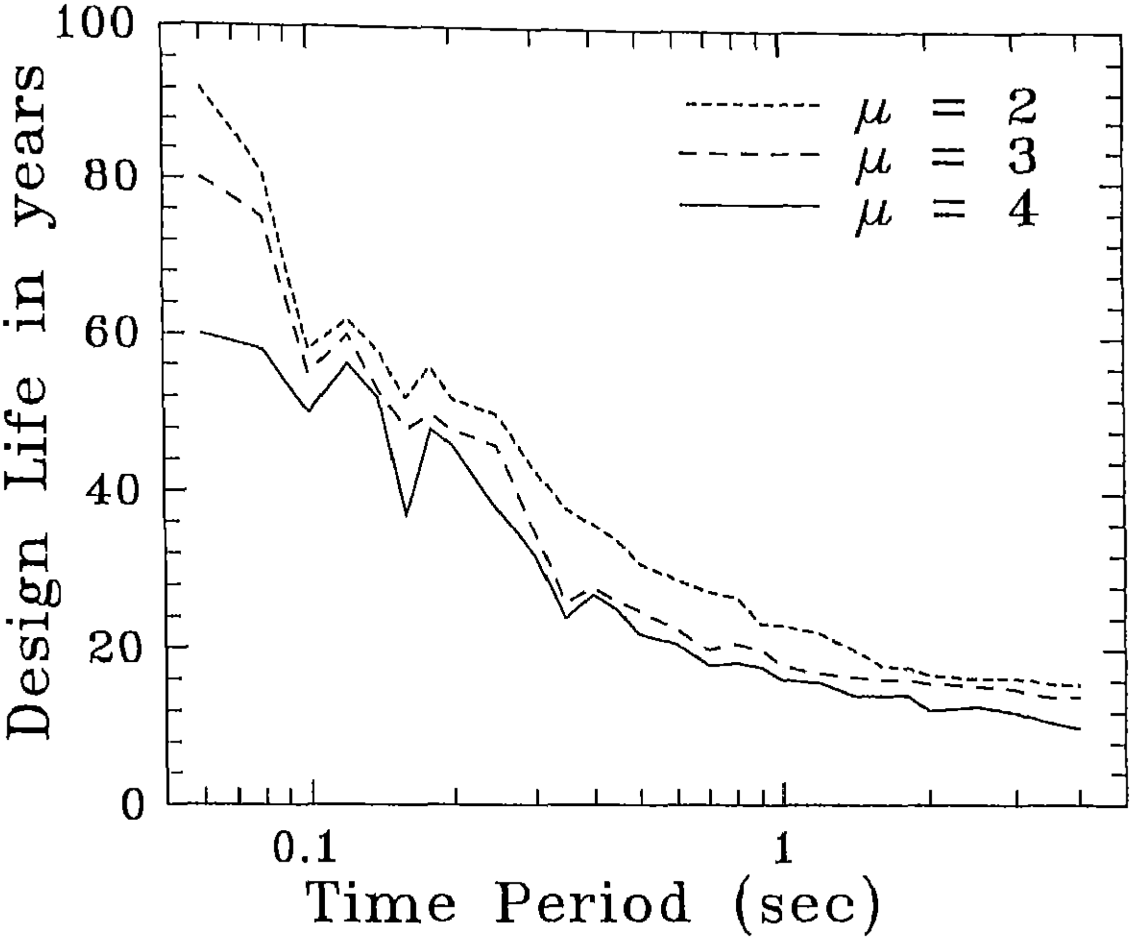

Figure 3.5 Design Life of a Set of Oscillators for $\mu = 2$, 3 and 4, with $\alpha_R = 2.0$ and $D = 0.8$.

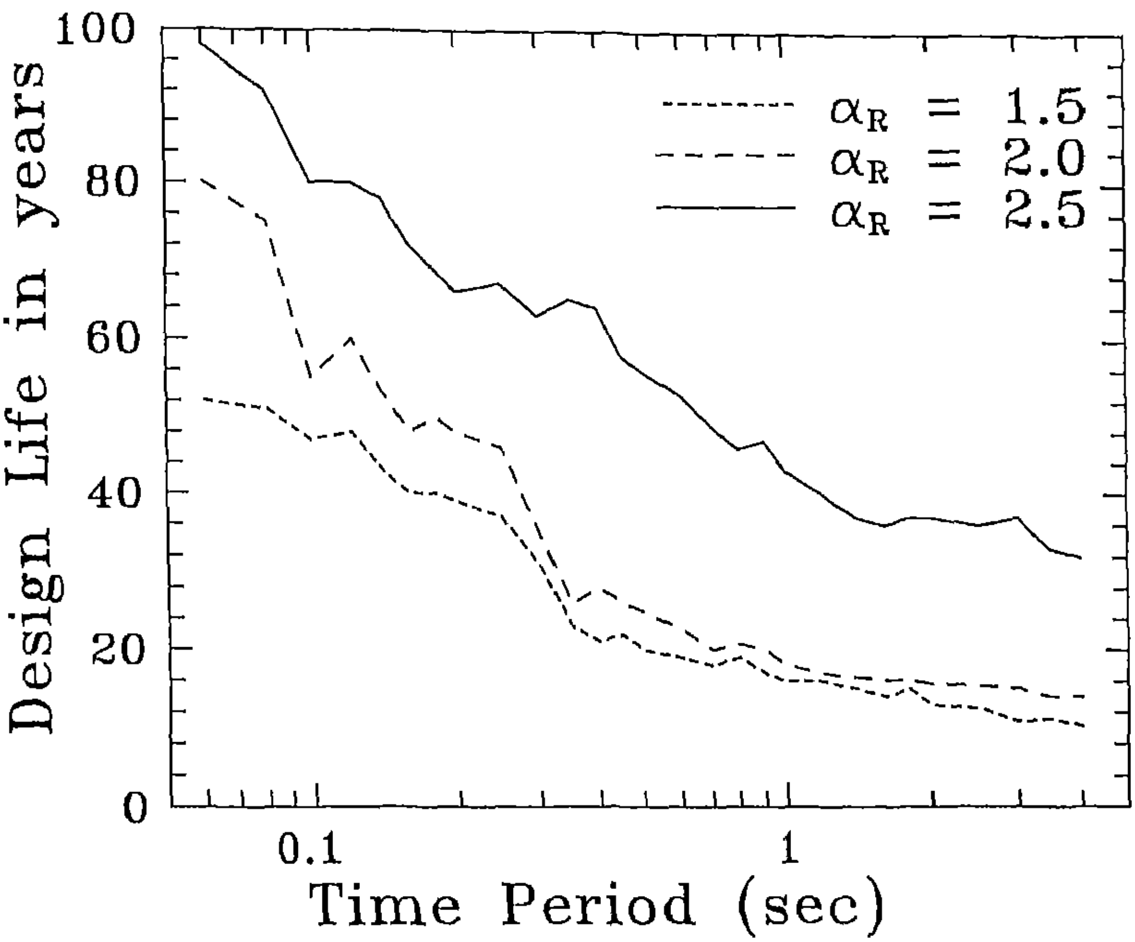

Figure 3.6 Design Life of a Set of Oscillators for $\alpha_R = 1.5$, 2.0 and 2.5, with $\mu = 3$ and $D = 0.8$.

more stiff oscillators, and thus, those are designed for higher design force levels. Fig 3 5 shows that with greater design ductility values, shorter design life should be considered for the designing of the structure This is because of the reason that by increasing ductility, we reduce the yield level of the structure which leads to greater number of inelastic excursions and thus to more damage Fig 3.6 shows that by choosing higher design strength levels, the structures can be designed for significantly greater design life values

Figs. 3.7 to 3.9 show the variations in the design ductility (with time period) respectively for varying design life, α_R, and maximum damage. Fig 3.7 shows that for a given oscillator, we can afford to have higher values of design ductility for the structures with smaller design life with the allowable damage remaining unchanged. Further, stiffer oscillators on average can have higher design ductility as compared to the flexible oscillators. It is seen in Figs. 3.8 and 3 9 that a higher value of α_R is compatible with greater design ductility for the same damage and that for a lower value of limiting damage, a lower design ductility should be considered.

Figs. 3.10 to 3.12 show how damage index depends on oscillator time period for varying design ductility, design life and α_R respectively. Consistent with the trends as observed above, damage increases with the natural period of the oscillator. It is higher in case of structures with greater design ductility and same design life, and for structures with greater design life and same design ductility. Similarly, damage is seen to decrease in Fig. 3.12 for increasing linear design level.

Figs. 3.13 to 3.15 show the variation of the minimum value of α_R for

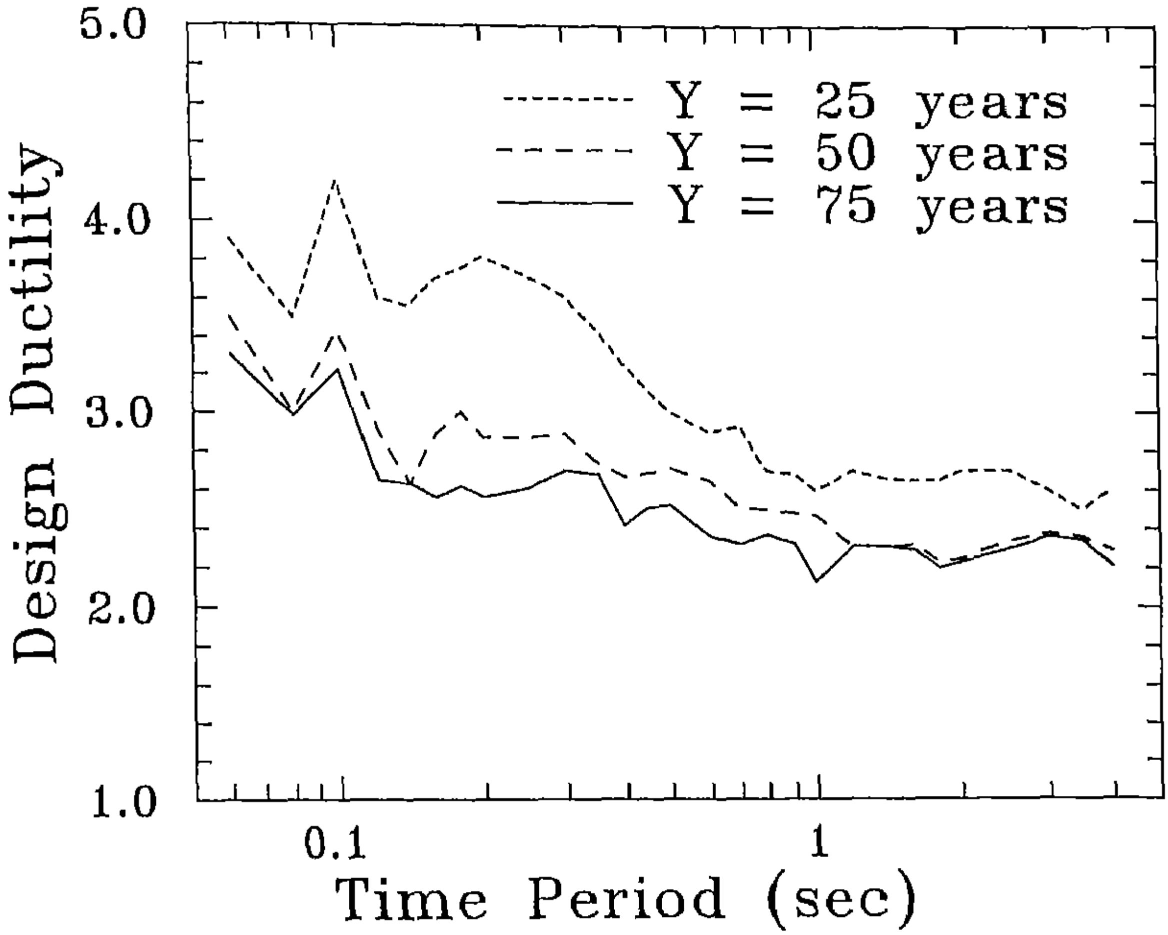

Figure 3.7 Design Ductility of a Set of Oscillators for Design Life, $Y = 25$, 50 and 75 years, with $\alpha_R = 2.0$ and $D = 0.8$.

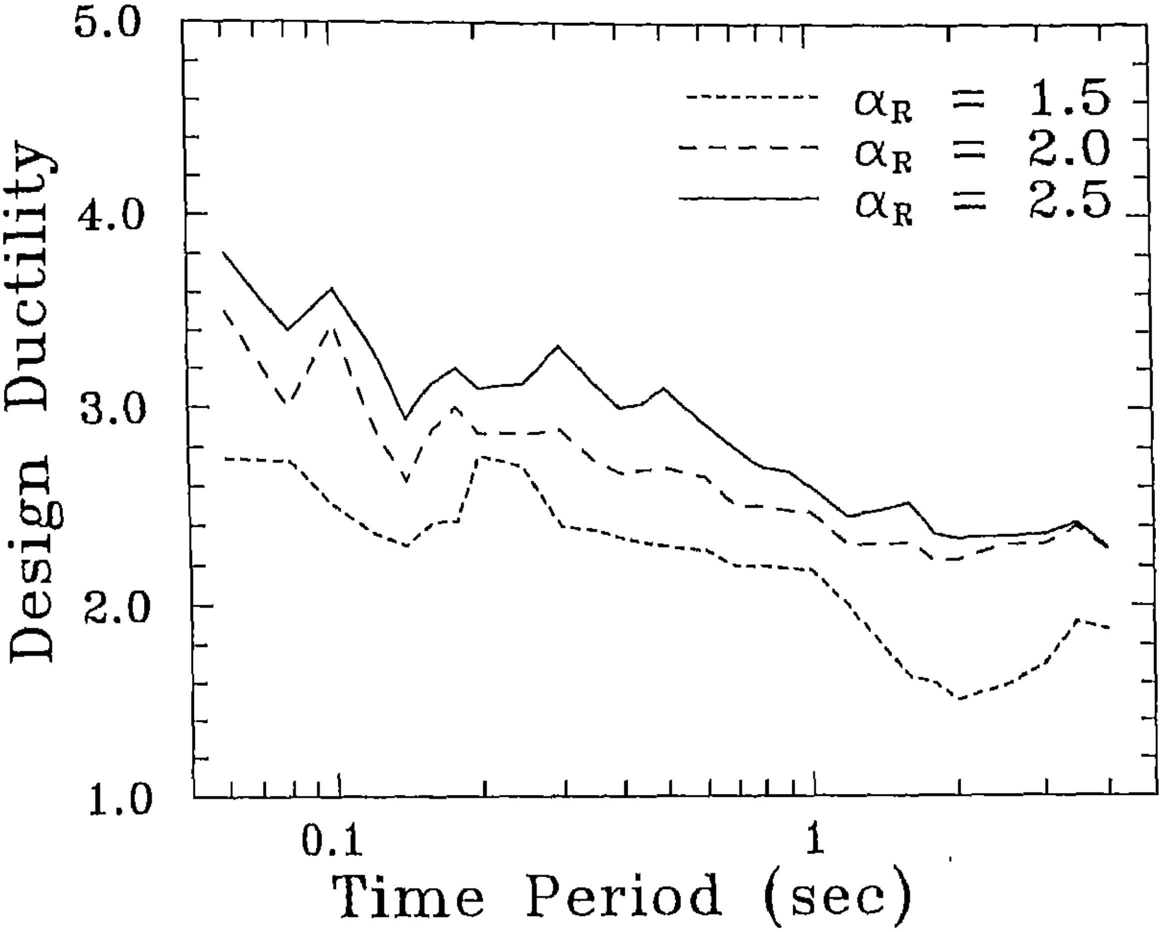

Figure 3.8 Design Ductility of a Set of Oscillators for $\alpha_R =$ 1.5, 2.0 and 2.5, with $Y = 50$ years and $D = 0.8$.

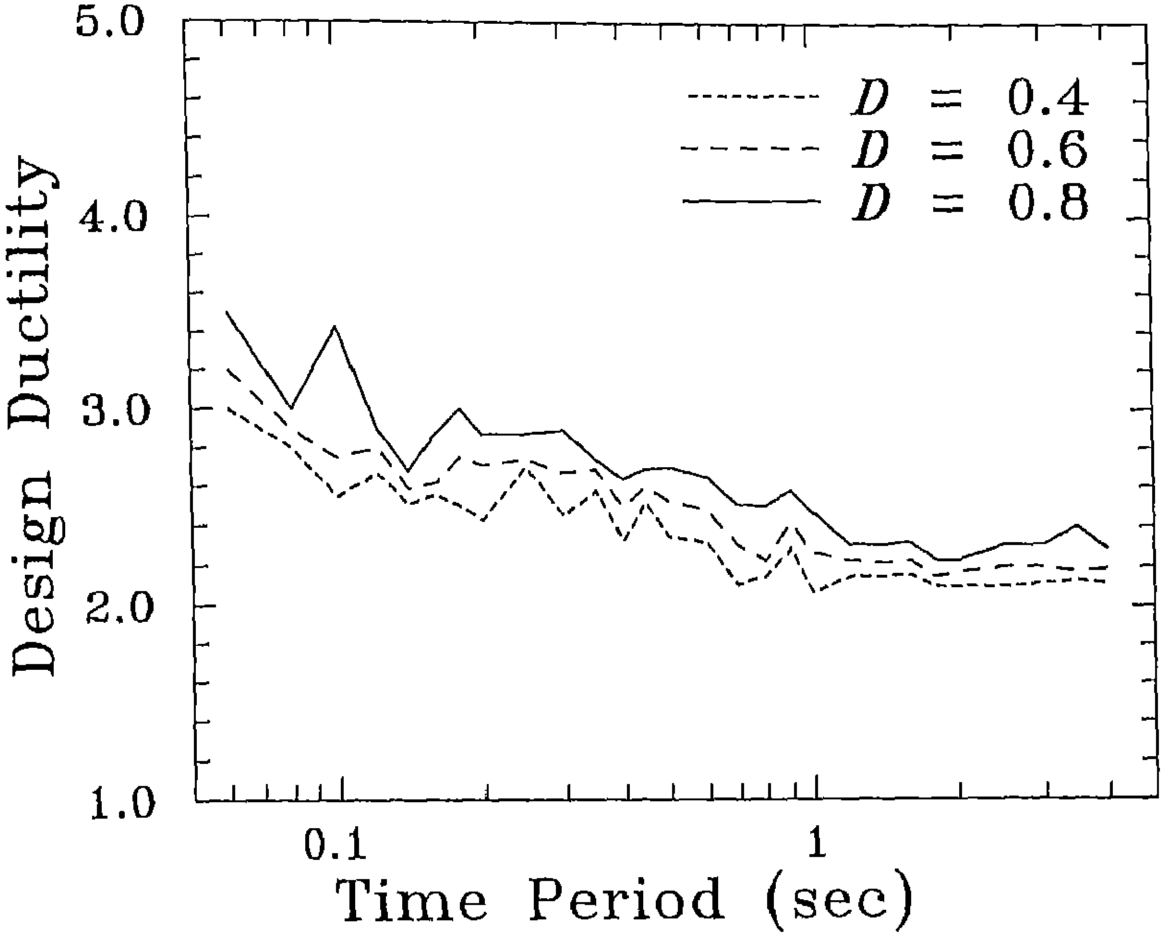

Figure 3.9 Design Ductility of a Set of Oscillators for an Allowable Damage, D = 0.4, 0.6 and 0.8, with Y = 50 years and $\alpha_R = 2.0$.

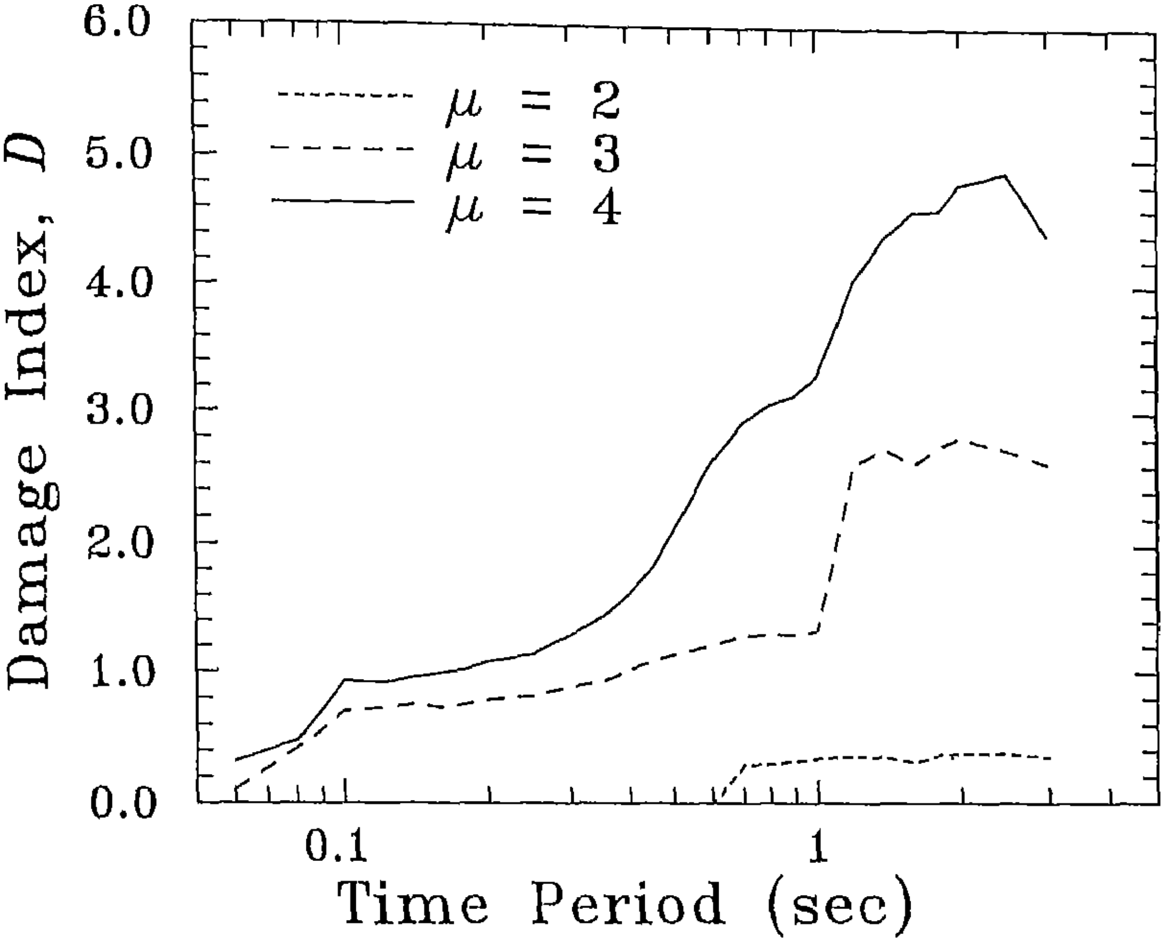

Figure 3.10 Damage Index of a Set of Oscillators for $\mu = 2$, 3 and 4, with $Y = 50$ years and $\alpha_R = 2$.

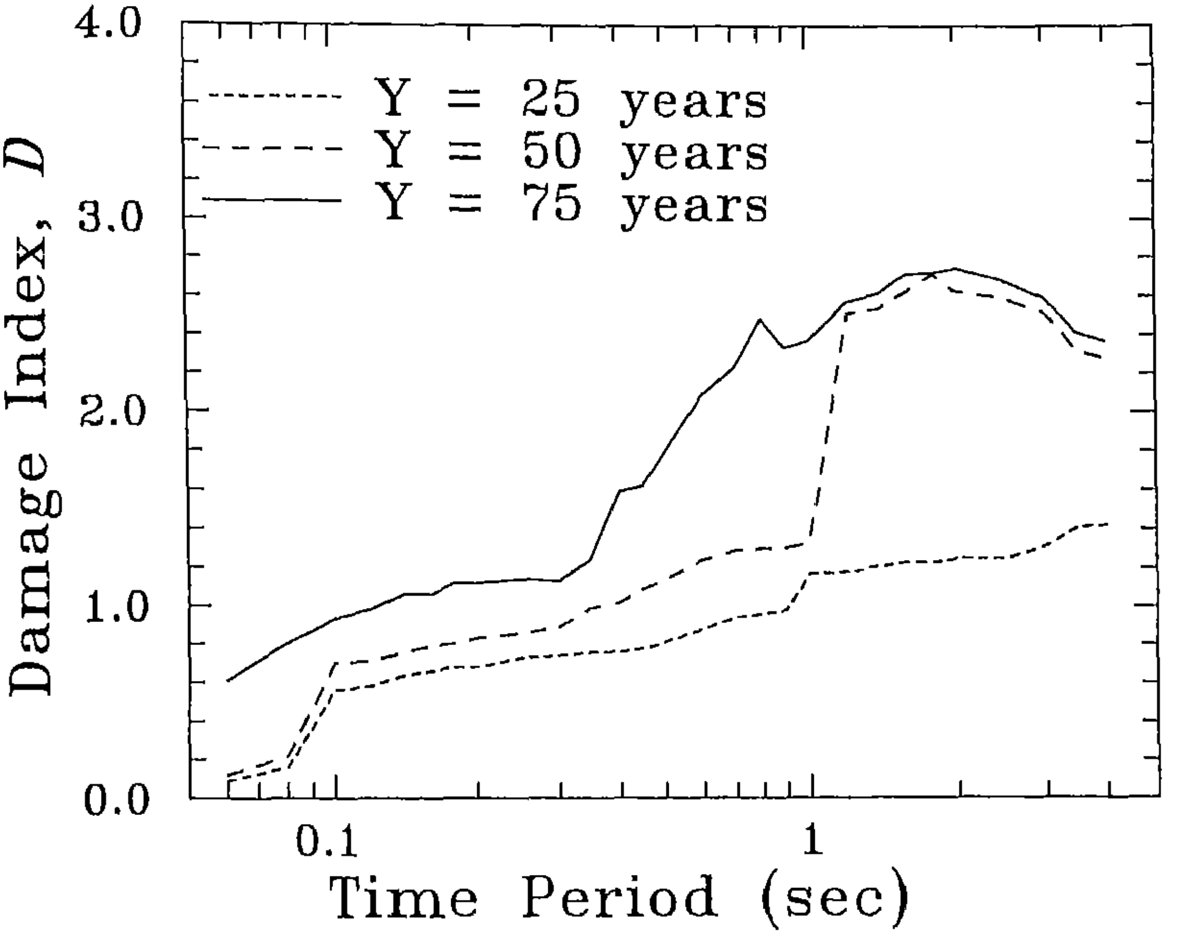

Figure 3.11 Damage Index of a Set of Oscillators for Design life, $Y = 25$, 50 and 75 years, with $\mu = 3$ and $\alpha_R = 2.0$.

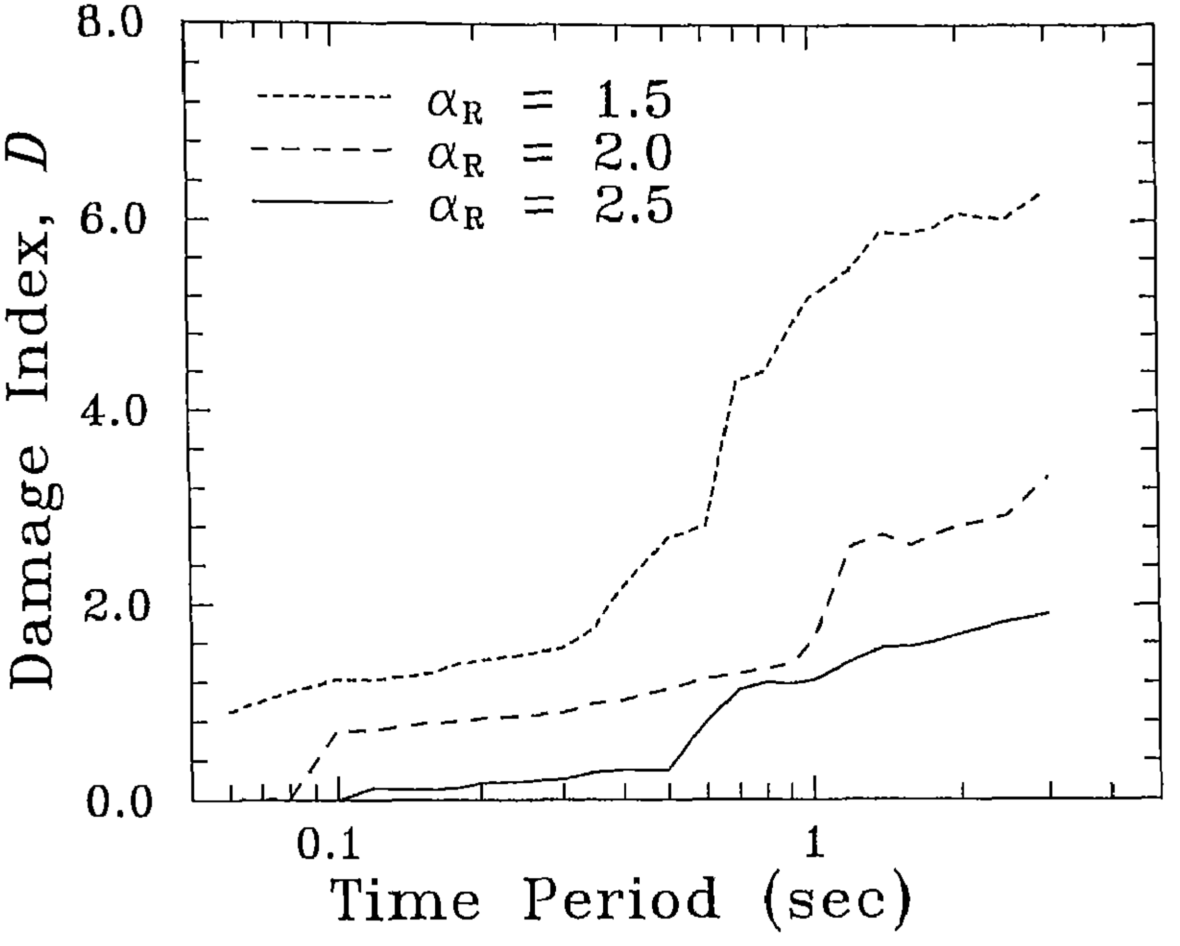

Figure 3.12 Damage Index of a Set of Oscillators for $\alpha_R = 1.5$, 2.0 and 2.5, with $Y = 50$ years and $\mu = 3$.

varying damage index, design ductility and design life respectively. All the three curves show the need to provide higher linear design levels for more flexible structural systems to have same damage in the same design life. Other trends are same as observed above, i.e., there is a need to provide higher linear design level than that required on the basis of response reduction factor, for i) limiting damage to lower levels, ii) greater design ductility, and iii) greater design life

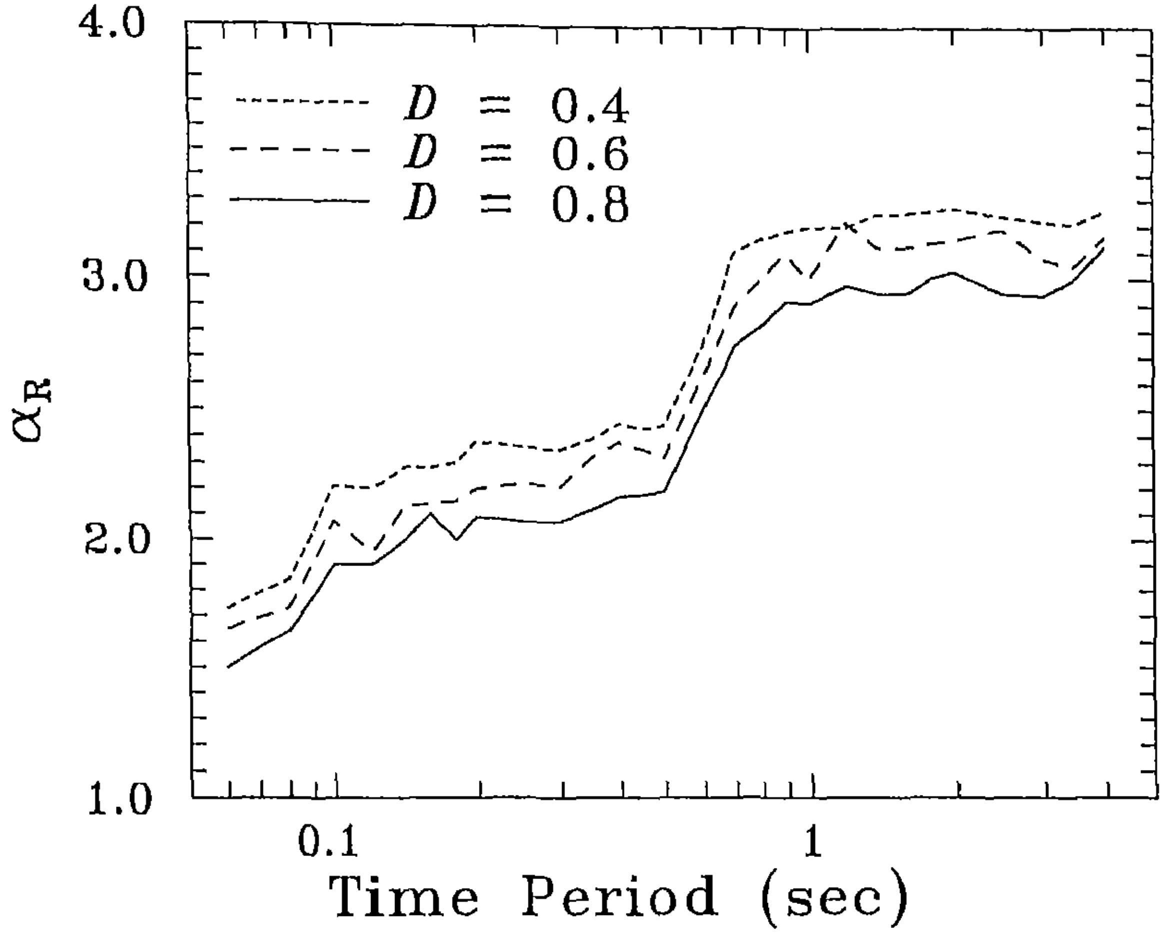

Figure 3.13 α_R for a Set of Oscillators with Allowable Damage, $D = 0.4$, 0.6 and 0.8, for $Y = 50$ years and $\mu = 3$.

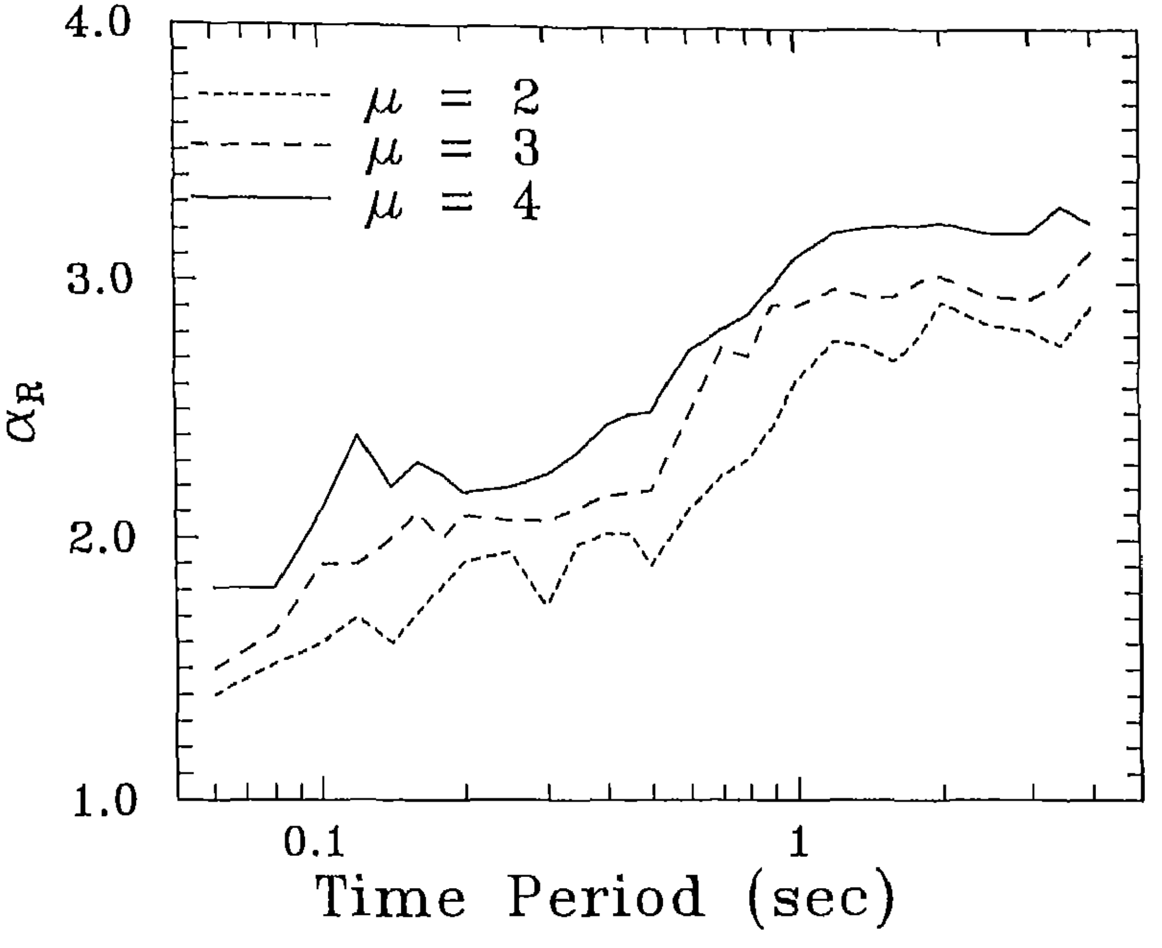

Figure 3.14 α_R for a Set of Oscillators with $\mu = 2$, 3 and 4, for $Y = 50$ years and $D = 0.8$.

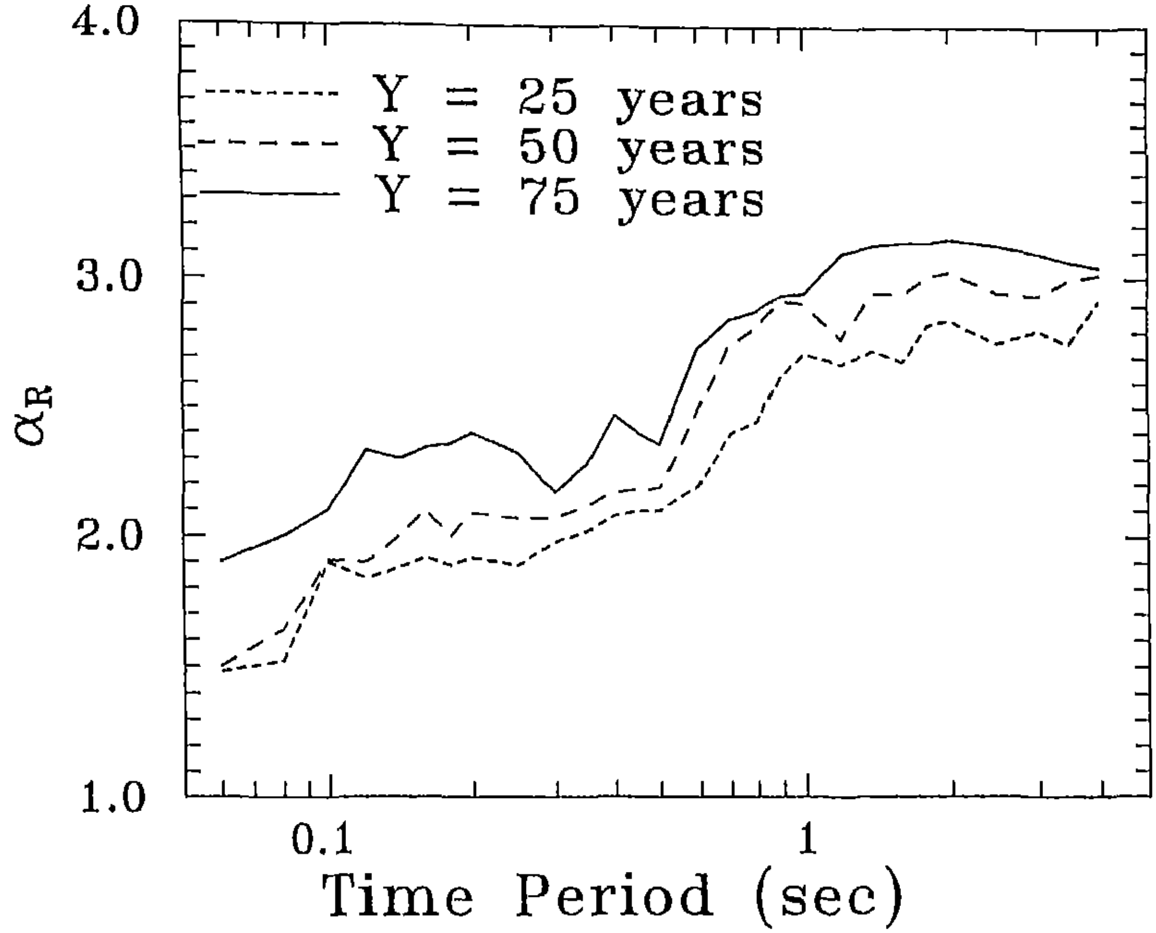

Figure 3.15 α_R for a Set of Oscillators with Design Life, $Y =$ 25 , 50 and 75 years for $D = 0.8$ and $\mu = 3$.

CHAPTER IV

CONCLUSIONS

A new approach has been proposed in this study for the estimation of the design life of a SDOF structure which is situated in a seismic environment The structure is expected to attain a specified level of allowable damage at the end of its design life. The proposed formulation is based on i) the time-dependent hazard model for the earthquake occurrences, ii) use of scaling equations for the estimation of ground motion during a given earthquake event, iii) linearizing the hysteretic SDOF oscillator by using the method of stochastic linearization, iv) use of modified Park and Ang's model and order statistics approach for calculation of the structural damage during each event, v) considering most critical sequencing of the earthquake events in a block of five years, and on vi) assuming a stiffness and strength degradation model for accounting for the degradation in the properties of the structure from one damaging event to another.

An illustrative study based on a hypothetical example has qualitatively led to the following important conclusions. It has been found that the 'single event-based' conventional method of design may be inappropriate for ensuring safety in those areas where, besides the most critical earthquake, several earthquakes of milder intensity may also occur during the design life of the structure For the usually adopted levels of force reduction from the linear levels, these earthquakes may generate sufficiently strong ground motions at the site of a structure so as to drive its response to be inelastic. Depending upon the damage

levels considered acceptable by the owners in view of the functional requirements of the structures, the reductions in the linear response levels should thus be much smaller and be not governed by the ductility of the system alone

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